Table 3-1 Damage Patterns and Hysteretic Response for Reinforced Masonry Components (continued)

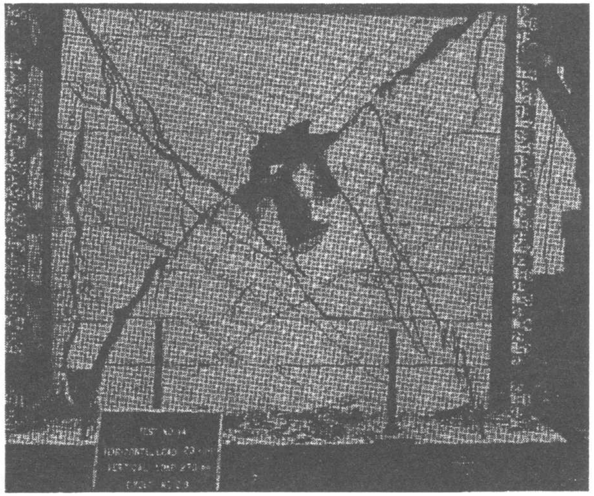
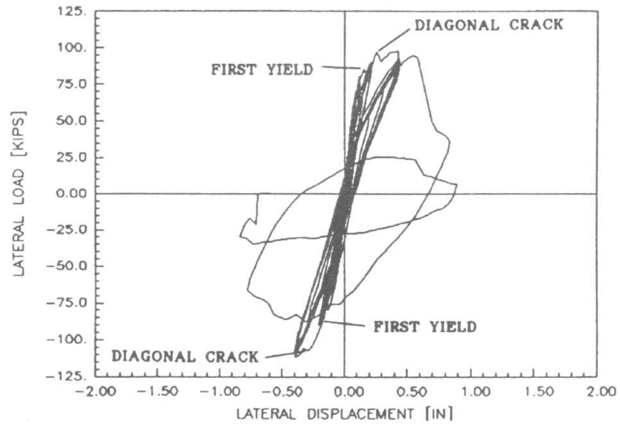
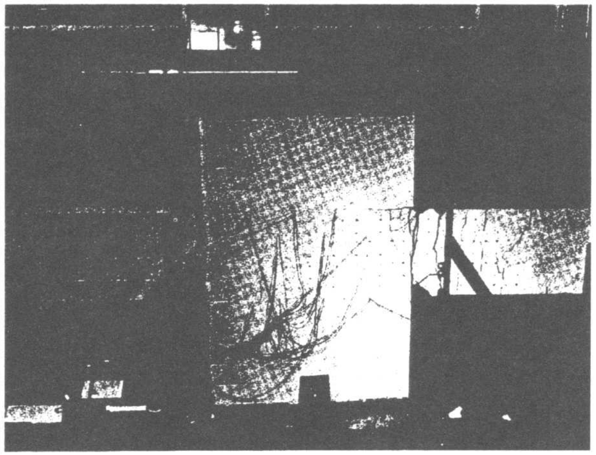
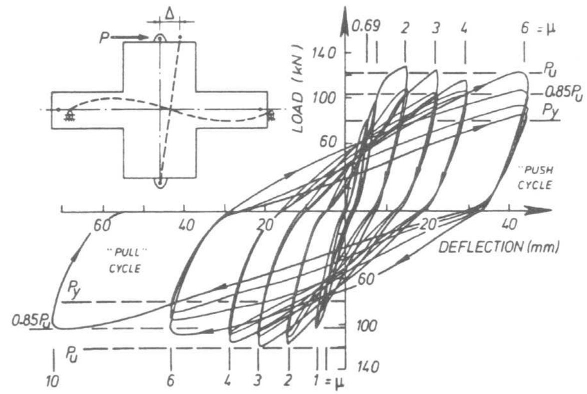
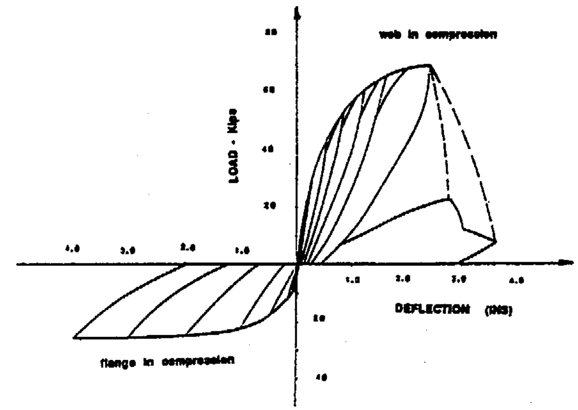
RM1 or RM2 Preemptive Shear *See Guide RM2G*	Shing et al., 1991 Specimen 14		
RM3 Flexure *See Guide RM3A*	Priestley and Hon, 1985		

Table 3-1 Damage Patterns and Hysteretic Response for Reinforced Masonry Components (continued)

RM1 or RM2 with flange. Flexure / Shear *See Guides RM1A, RM1B, and / or RM2G*	Priestley and He, 1990		

Research has been conducted to evaluate the relationship between crack width, crack spacing, and reinforcing bar strain. A partial review of the literature on crack width is provided by Noakowski, (1985). Research indicates that the width of a crack crossing a reinforcing bar at first yield of the reinforcement depends on the bar diameter, the reinforcement yield stress, the reinforcement ratio, the reinforcement elastic modulus, and on the characteristics of the bond stress-slip relationship. However, most research in this area has focused on nearly elastic systems (prior to yield in reinforcement), and flexural cracking in beams and uniaxial tension specimens. It is difficult to extrapolate quantitative expressions for crack width and spacing prior to yield to reinforced masonry specimens with sufficient damage to reduce strength or deformation capacity.

Sassi and Ranous (1996) have suggested criteria to relate crack width to damage, but they have not provided sufficient information to associate crack patterns with specific behavior modes, which is essential when determining damage severity.

In the guides for reinforced masonry components, the crack width limits for each damage severity level have been determined empirically, using crack widths reported in the literature and photographs of damaged specimens. Consideration has been given to the theoretical crack width required to achieve yield of reinforcement under a variety of conditions. A fundamental presumption is that the width of shear cracks is related to damage severity, while flexural crack widths are not closely related to damage severity.

3.1.3 Interpretation of Tests

Interpretation of test results for reinforced masonry was similar to that for reinforced concrete as described in Section 2.1.1.2. The ranges of component ductility and l-factors are presented in Table 3-2.

3.2 Tabular Bibliography for Reinforced Masonry

Table 3-3 contains a brief description of the key technical reports which address specific reinforced masonry component behavior. The component types and their behavior modes are indicated. The full references can be found in Section 3.4.

Table 3-2 ***Ranges of reinforced masonry component displacement ductility, μ_Δ, associated with damage severity levels and λ factors***

Damage Guide	Damage Severity			
	Insignificant	**Slight**	**Moderate**	**Heavy**
RM1A Ductile Flexural	$\mu_\Delta \leq 3$ $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 2 - 4$ $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 3 - 8$ $\lambda_K = 0.4$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	*Heavy* not used
RM1B Flexure/Shear	$\mu_\Delta \leq 2$ $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 2 - 3$ $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 3 - 5$ $\lambda_K = 0.4$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	
RM1C Flexure/ Sliding Shear	See RM1A	$\mu_\Delta \approx 2 - 4$ $\lambda_K = 0.5$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	$\mu_\Delta \approx 3 - 8$ $\lambda_K = 0.2$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	
RM1D Flexure/ Out-of-Plane Instability	See RM1A	See RM1A	See RM1A	$\mu_\Delta \approx 8 - 10$ $\lambda_K = 0.4$ $\lambda_Q = 0.5$ $\lambda_D = 0.5$
RM1E Flexure/ Lap Splice Slip	See RM1A or RM1B	See RM1A or RM1B	$\mu_\Delta \approx 3 - 4$ $\lambda_K = 0.4$ $\lambda_Q = 0.5$ $\lambda_D = 0.8$	
RM2B Flexure/Shear	$\mu_\Delta \leq 2$ $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 2 - 3$ $\lambda_K = 0.6$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 3 - 5$ $\lambda_K = 0.4$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	*Heavy* not used
RM2G Preemptive Shear	$\mu_\Delta \leq 1$ $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 1 - 2$ $\lambda_K = 0.8$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \approx 1 - 2$ $\lambda_K = 0.5$ $\lambda_Q = 0.8$ $\lambda_D = 0.9$	$\mu_\Delta \approx 2 - 3$ $\lambda_K = 0.3$ $\lambda_Q = 0.4$ $\lambda_D = 0.5$
RM3A Flexure	$\mu_\Delta \leq 2$ $\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\mu_\Delta \leq 3$ $\lambda_K = 0.8$ $\lambda_Q = 0.9$ $\lambda_D = 1.0$	$\mu_\Delta \approx 6$ $\lambda_K = 0.6$ $\lambda_Q = 0.8$ $\lambda_D = 1.0$	
RM3G Preemptive Shear (No μ values for RM3G)	$\lambda_K = 0.9$ $\lambda_Q = 1.0$ $\lambda_D = 1.0$	$\lambda_K = 0.8$ $\lambda_Q = 0.8$ $\lambda_D = 1.0$	$\lambda_K = 0.3$ $\lambda_Q = 0.5$ $\lambda_D = 0.9$	

Table 3-3 Annotated Bibliography for Reinforced Masonry

Reference(s)	Description	Comp. Type(s)	Behavior modes Addressed a	b	c	d	e	f	g
EVALUATION AND DESIGN RECOMMENDATIONS									
Paulay and Priestley (1992)	Overview of capacity-design principles for reinforced concrete and masonry structures. Thorough description of R/C failure modes, and, to a lesser extent, R/M failure modes. Description of R/M component response in terms of displacement and ductility.	RM1 RM2 RM3 RM4	•	•	•	•	•	•	•
OVERVIEWS OF EXPERIMENTAL TEST RESULTS									
Drysdale, Hamid, and Baker (1994)	Textbook for design of masonry structures. Includes complete bibliography and selected results from experimental research.	RM1 RM2 RM4	•	•					•
EXPERIMENTAL TEST RESULTS									
Abrams and Paulson (1989) Abrams and Paulson (1990)	<u>2 specimens</u> 1/4-scale model	RM2	•	•	•				
Foltz and Yancy (1993)	<u>10 Specimens</u> 8" CMU 56" tall by 48" wide Axial load 200+ psi No vertical reinforcement $\rho_v = 0.0\%$ $\rho_h = 0.024\% - 0.22\%$ Axial load increased w/ displacement. Clear improvement in displacement and crack distribution w/ increased horizontal reinforcement. Many damage photos. No hysteresis curves. Joint reinforcement improved ultimate displacement from μ=1 to μ=3.	RM2							•

Table 3-3 Annotated Bibliography for Reinforced Masonry (continued)

Ghanem et al. (1993)	<u>14 Specimens</u> 1/3 scale concrete block	Monotonic tests only reported here.		RM2							•
Hammons et al. (1994)	<u>124 specimens</u> Hollow concrete and clay masonry	Monotonic testing of lap splices. Only #4 in 8" units fail by classical pull-out. Others fail in tensile splitting.	Tensile splitting failure likely regardless of lap splice length for: #4 in 4 inch units #6 in 6 inch units #8 in 8 inch units	N/A							
Hidalgo et al. (1978) Chen et al. (1978) Hidalgo et al. (1979)	<u>63 specimens:</u> 28 8" hollow clay brick 18 2-wythe clay brick 17 8" hollow concrete block	Aspect ratios: 0.5, 1.0, 2.0 High axial loads, increasing with lateral displacement.	All failures in shear or flexure/ shear	RM2		•					•
Hon & Priestley (1984) Priestley & Hon (1985) Hart & Priestley (1989) Priestley (1990)	<u>2 fully-grouted specimens</u> 8" hollow concrete block One specimen tested in New Zealand, and a second later at UC San Diego.	Full-scale, fully-reversed cyclic loading. 2nd specimen purposely violated proposed design criteria, and performed in a ductile manner.	Stable hysteresis up to displacement ductility of 4 at first crushing. Achieved ductility of 10 with minor load degradation.	RM3	•						
Igarashi et al. (1993)	<u>1 fully grouted 3-story wall specimen</u> 6" hollow concrete block 3-story full-scale cantilever wall	$\rho_v = 0.15\%$ $\rho_h = 0.22\%$	Flexural response to 0.3% drift followed by lap-splice slip at base and stable rocking to 1% drift at approx. 1/3 of max. load.	RM1					•		
Kubota and Murakami (1988)	<u>5 cmu wall specimens</u> Investigated effect of lap splices	Sudden loss of strength associated w/ lap-splice failure. Test stopped following lap-splice failure	Vertical splitting at lap	RM2		•			•		
Kubota et al. (1985)	<u>5 wall specimens</u> Hollow clay brick	Minimum vertical reinf $\rho_h = 0.17\% - 0.51\%$		RM2		•					•
Matsumura (1988)	Includes effect of grout flaws on damage patterns and shear strength.	Missing or insufficient grout causes localized damage and inhibits uniform distribution of cracks.		RM2		•					•

Table 3-3 Annotated Bibliography for Reinforced Masonry (continued)

Matsuno et al. (1987)	<u>1 grouted hollow clay specimen</u> 3-stories 3-coupled flanged walls	Limited ductility, significant strength degradation associated w/ preemptive shear failure of coupling beams.	Flexure response in long wall (RM1) Flexure/shear in short walls (RM2)	RM1 RM2 RM4		•			•		•
Merryman et al (1990) Leiva and Klingner (1991)	<u>6 fully-grouted, 2-story wall specimens</u> 2-story walls with openings 2-story pairs of wall coupled by slab only 2-story pairs of walls coupled by slab and R/M lintel	Flexural design by 1985 UBC. Shear design to ensure flexure hinging. $\rho_v = 0.22\%$ $\rho_h = 0.22\% - 0.44\%$	Stable flexural response in coupled walls, limited by compression toe spalling, fracture of reinforcement, and sliding. No significant load degradation even at end of test. One specimen inadvertently loaded to 60% of max base shear in single pulse prior to test, with no clear effect on response.	RM1	•		•				
Okada and Kumazawa (1987)	Concrete block beams 32"x90"	Similar to concrete. Rotation capacity of 1/100	Damage for lap splices limited to splice zone. More distributed without laps.	RM4	•	•					
Priestley and Elder (1982)				RM1	•	•					
Schultz, (1996)	<u>6 partially-grouted specimens</u> concrete masonry	Minimum vertical reinf $\rho_h = .05\% - .12\%$ Moderately ductile response w/ initial peak and drop to degrading plateau at approx. 75% of max.	Drift = 0.3%-1% at 75% of max strength. Behavior characterized by vertical cracks at junction of grouted and ungrouted cells. Few if any diagonal cracks except in one specimen.	RM2		•					

Table 3-3 Annotated Bibliography for Reinforced Masonry (continued)

Seible et al. (1994) Seible et al. (1995) Kingsley (1994) Kingsley et al. (1994) Kürkchübasche et al. (1994)	<u>1 fully grouted, 5-story building specimen</u> 6" hollow concrete block 5-story full-scale flanged walls coupled by topped, precast plank floor system $\rho_v = 0.23\%\text{-}0.34\%$ $\rho_h = 0.11\% - 0.44\%$	Flexural design by 1991 *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings.* Shear design to ensure flexural hinging.	Ductile flexural response with some sliding to μ=6 and 9, (drift = 1% and 1.5%). Distributed cracking. Significant influence of flanges and coupling slabs.	RM1	•		•				
Shing et al. (1990a) Shing et al. (1990b) Shing et al. (1991)	<u>24 fully-grouted test specimens</u>: 6 6-inch hollow clay brick 18 6-inch hollow concrete block 2 monotonic loading 22 cyclic-static loading. 4 levels of axial load	Full-scale walls, 6-ft square, loaded in single curvature. $M/VL = 1$ Uniformly distributed vertical & horizontal reinforcement. $\rho_v = 0.38\% - 0.74\%$ $\rho_h = 0.14\% - 0.26\%$	2 specimens with lap splices at base, others with continuous reinforcement. 1 specimen w/ confinement comb at wall toe. Most comprehensive tests on reinforced masonry wall components to date	RM1 RM2	•	•	•				•
Tomazevic and Zarnic (1985) Tomazevic and Lutman (1988) Tomazevic and Modena (1988) Tomazevic et al. (1993)	<u>32 wall specimens</u> Concrete block walls and complete structures Static and shaking table	$\rho_v = 0.26\% - 0.52\%$ $\rho_h = 0.00\% - 0.52\%$		RM2	•	•					•
Yamazaki et al. (1988a) Yamazaki et al. (1988b)	<u>1 fully-grouted 5-story building specimen</u> 8" hollow concrete block 5-story full-scale flanged walls coupled by cast-in-place 6" and 8" R/C floor slabs	First damage in masonry lintel beams of many different geometries.	Flexural modes degraded to shear failing modes at 0.75% building drift (1.4% first story drift).	RM1 RM2 RM4		•					•

Table 3-3 Annotated Bibliography for Reinforced Masonry (continued)

EXPERIMENTAL TEST RESULTS – REPAIRED OR RETROFITTED WALLS											
Innamorato (1994)	<u>3 fully-grouted test specimens</u> Designed to match Shing (1991) Preemptive shear failure Flexure failure	Tested in "original" and "repaired" condition	Repair by epoxy injection and carbon fiber overlay	RM1 RM2	•	•					•
Laursen et al. (1995)	<u>2 in-plane specimens</u> Designed to match Shing (1991) specimen preemptive shear failure. <u>2 out-of-plane specimens</u>	Tested in "original," "repaired," and "retrofit" configurations.	Repair by epoxy injection and carbon fiber overlays in horizontal or vertical direction to enhance ductility or strength	RM1 RM2	•						•
Weeks et al. (1994)	5-story building tested previously by Seible et al. (1994) repaired and retested.		Repair by epoxy injection and carbon fiber overlay	RM1							

[1] <u>Behavior modes:</u>

a Ductile Flexural Response:

b Flexure/Diagonal Shear

c Flexure/Sliding Shear

d Flexure/Out-of-Plane Wall Buckling

e Flexure/Lap-Splice Slip

f Foundation rocking of individual piers

g Preemptive Diagonal Shear Failure

3.3 Symbols for Reinforced Masonry

A_g = Gross crossectional area of wall

A_{si} = Area of reinforcing bar i

A_v = Area of shear reinforcing bar

A_{vf} = Area of reinforcement crossing perpendicular to the sliding plane

a = Depth of the equivalent stress block

c = Depth to the neutral axis

C_m = Compression force in the masonry

f_{me} = Expected compressive strength of masonry

f_{ye} = Expected yield strength of reinforcement

h_e = Effective height of the wall (height to the resultant of the lateral force) = M/V

l_d = Lap splice development length

l_p = Effective plastic hinge length

l_w = Length of the wall

M/V = Ratio of moment to shear (shear span) at a section

M_e = Expected moment capacity of a masonry section

P_u = Wall axial load

s = Spacing of reinforcement

t = Wall thickness

V_e = Expected shear strength of a reinforced masonry wall

V_m = Portion of the expected shear strength of a wall attributed to masonry

V_s = Portion of the expected shear strength of a wall attributed to steel

V_p = Portion of the expected shear strength of a wall attributed to axial compression effects

V_{se} = Expected sliding shear strength of a masonry wall

x_i = Location of reinforcing bar i

Δ_p = Maximum inelastic displacement capacity

Δ_y = Displacement at first yield

ϕ_m = Maximum inelastic curvature of a masonry section

ϕ_y = Yield curvature of a masonry section

μ_Δ = Displacement ductility

μ = Coefficient of friction at the sliding plane

3.4 References for Reinforced Masonry

This list contains references from the reinforced masonry chapters of both FEMA 306 and 307.

Abrams, D.P., and Paulson, T.J., 1989, "Measured Nonlinear Dynamic Response of Reinforced Concrete Masonry Building Systems," *Proceedings of the Fifth Canadian Masonry Symposium*, University of British Columbia, Vancouver, B.C., Canada.

Abrams, D.P., and Paulson, T.J., 1990, "Perceptions and Observations of Seismic Response for Reinforced Masonry Building Structures," *Proceedings of the Fifth North American Masonry Conference*, University of Illinois at Urbana-Champaign.

Agbabian, M., Adham, S, Masri, S.,and Avanessian, V., *Out-of-Plane Dynamic Testing of Concrete Masonry Walls*, U.S. Coordinated Program for Masonry Building Research, Report Nos. 3.2b-1 and 3.2b-2.

Anderson, D.L., and Priestley, M.J.N., 1992, "In Plane Shear Strength of Masonry Walls," *Proceedings of the 6th Canadian Masonry Symposium*, Saskatoon, Saskatchewan.

Atkinson, R.H.,Amadei, B.P.,Saeb, S., and Sture, S., 1989, "Response of Masonry Bed Joints in Direct Shear," *American Society of Civil Engineers Journal of the Structural Division*, Vol. 115, No. 9.

Atkinson, R.H., and Kingsley, G.R., 1985, *A Comparison of the Behavior of Clay and Concrete Masonry in Compression*, U.S. Coordinated Program for Masonry Building Research, Report No. 1.1-1.

Atkinson, R.H., Kingsley, G.R., Saeb, S., B. Amadei, B., and Sture, S., 1988, "A Laboratory and In-situ Study of the Shear Strength of Masonry Bed Joints," *Proceedings of the 8th International Brick/ Block Masonry Conference*, Dublin.

BIA, 1988, *Technical Notes on Brick Construction, No. 17*, Brick Institute of America, Reston, Virginia.

Blakeley, R.W.G., Cooney, R.C., and Megget, L.M., 1975, "Seismic Shear Loading at Flexural Capacity in Cantilever Wall Structures," *Bulletin of the New Zealand National Society for Earthquake Engineering*, Vol. 8, No. 4.

Calvi, G.M., Macchi, G., and Zanon, P., 1985, "Random Cyclic Behavior of Reinforced Masonry Under Shear Action," *Proceedings of the Seventh International Brick Masonry Conference*, Melbourne, Australia.

Chen, S.J., Hidalgo, P.A., Mayes, R.L., and Clough, R.W., 1978, *Cyclic Loading Tests of Masonry Single Piers, Volume 2 – Height to Width Ratio of 1*, Earthquake Engineering Research Center Report No. UCB/EERC-78/28, University of California, Berkeley, California.

Drysdale, R.G., Hamid, A.A., and Baker, L.R., 1994, *Masonry Structures, Behavior and Design*, Prentice Hall, New Jersey.

Fattal, S.G., 1993, *Strength of Partially-Grouted Masonry Shear Walls Under Lateral Loads*, National Institute of Standards and Technology, NISTIR 5147, Gaithersburg, Maryland.

Foltz, S., and Yancy, C.W.C., 1993, "The Influence of Horizontal Reinforcement on the Shear Performance of Concrete Masonry Walls", *Masonry: Design and Construction, Problems and Repair*, ASTM STP 1180, American Society for Testing and Materials, Philadelphia, Pennsylvania.

Ghanem, G.M., Elmagd, S.A., Salama, A.E., and Hamid, A.A., 1993, "Effect of Axial Compression on the Behavior of Partially Reinforced Masonry Shear Walls," *Proceedings of the Sixth North American Masonry Conference*, Philadelphia, Pennsylvania.

Hamid, A., Assis, G., and Harris, H., 1988, *Material Models for Grouted Block Masonry,* U.S. Coordinated Program for Masonry Building Research, Report No. 1.2a-1.

Hamid, A., Abboud, B., Farah, M., Hatem, K., and Harris, H., 1989, *Response of Reinforced Block Masonry Walls to Out-of-Plane Static Loads,* U.S. Coordinated Program for Masonry Building Research, Report No. 3.2a-1.

Hammons, M.I., Atkinson, R.H., Schuller, M.P.,and Tikalsky, P.J., 1994, *Masonry Research for Limit-States Design, Construction Productivity Advancement Research (CPAR)*U.S. Army Corps of Engineers Waterways Experiment Station, Program Report CPAR-SL-94-1, Vicksburg Mississippi.

Hanson, R.D., 1996, "The Evaluation of Reinforced Concrete Members Damaged by Earthquakes", *Earthquake Spectra*, Vol. 12, No. 3, Earthquake Engineering Research Institute.

Hart, G.C., and Priestley, M.J.N., 1989, *Design Recommendations for Masonry Moment-Resisting Wall Frames*, UC San Diego Structural Systems Research Project Report No. 89/02.

Hart, G.C., Priestley, M.J.N., and Seible, F., 1992, "Masonry Wall Frame Design and Performance," *The Structural Design of Tall Buildings*, John Wile & Sons, New York.

He, L., and Priestley, M.J.N., 1992, *Seismic Behavior of Flanged Masonry Walls*, University of California, San Diego, Department of Applied Mechanics and Engineering Sciences, Report No. SSRP-92/09.

Hegemier, G.A., Arya, S.K., Nunn, R.O., Miller, M.E., Anvar, A., and Krishnamoorthy, G., 1978, *A Major Study of Concrete Masonry Under Seismic-Type Loading*, University of California, San Diego Report No. AMES-NSF TR-77-002.

Hidalgo, P.A., Mayes, R.L., McNiven, H.D., and Clough, R.W., 1978, *Cyclic Loading Tests of Masonry Single Piers, Volume 1 – Height to Width Ratio of 2*, University of California, Berkeley, Earthquake Engineering Research Center Report No. UCB/EERC-78/27.

Hidalgo, P.A., Mayes, R.L., McNiven, H.D., and Clough, R.W., 1979, *Cyclic Loading Tests of Masonry Single Piers, Volume 3 – Height to Width Ratio of 0.5*, University of California, Berkeley, Earthquake Engineering Research Center Report No. UCB/EERC-79/12.

Hon, C.Y., and Priestley, M.J.N., 1984, *Masonry Walls and Wall Frames Under Seismic Loading*, Department of Civil Engineering, University of Canterbury, Research Report 84-15, New Zealand.

Igarashi, A., F. Seible, and Hegemier, G.A., 1993, *Development of the Generated Sequential Displacement Procedure and the Simulated Seismic Testing of the TCCMAR 3-Story In-plane Walls*, U.S. Coordinated Program for Masonry Building Research Report No. 3.1(b)-2.

Innamorato, D. , 1994, *The Repair of Reinforced Structural Masonry Walls using a Carbon Fiber, Polymeric Matrix Composite Overlay*, M.Sc. Thesis in Structural Engineering, University of California, San Diego.

Kariotis, J.C., Rahman, M. A., El-mustapha, A. M., 1993, "Investigation of Current Seismic Design Provisions for Reinforced Masonry Shear Walls," *The Structural Design of Tall Buildings*, Vol. 2, 163-191.

Kingsley, G.R., 1994, *Seismic Design and Response of a Full-scale Five-Story Reinforced Masonry Research Building*, Doctoral Dissertation, University of California, San Diego, School of Structural Engineering.

Kingsley, G.R., Noland, J.L., and Atkinson, R.H., 1987, "Nondestructive Evaluation of Masonry Structures Using Sonic and Ultrasonic Pulse Velocity Techniques," *Proceedings of the Fourth North American Masonry Conference*, University of California at Los Angeles.

Kingsley, G.R., Seible, F., Priestley, M.J.N., and Hegemier, G.A., 1994, *The U.S.-TCCMAR Full-scale Five-Story Masonry Research Building Test, Part II: Design, Construction, and Test Results*, Structural Systems Research Project, Report No. SSRP-94/02, University of California at San Diego.

Kubota, T., and Murakami, M., 1988, "Flexural Failure Test of Reinforced Concrete Masonry Walls -- Effect of Lap Joint of Reinforcement," *Proceedings of the Fourth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research*, San Diego, California.

Kubota, T., Okamoto, S., Nishitani, Y., and Kato, S., 1985, "A Study on Structural Behavior of Brick Panel Walls," *Proceedings of the Seventh International Brick Masonry Conference*, Melbourne, Australia.

Kürkchübasche, A.G., Seible, F., and Kingsley, G.R., 1994, *The U.S.-TCCMAR Five-story Full-scale Reinforced Masonry Research Building Test: Part IV, Analytical Models*, Structural Systems Research Project, Report No. TR - 94/04, University of California at San Diego.

Laursen, P. T., Seible, F., and Hegemier, G. A. , 1995, *Seismic Retrofit and Repair of Reinforced Concrete with Carbon Overlays,* Structural Systems Research Project, Report No. TR - 95/01, University of California at San Diego.

Leiva, G., and Klingner, R.E., 1991, *In-Plane Seismic Resistance of Two-story Concrete Masonry Shear Walls with Openings,* U.S. Coordinated Program for Masonry Building Research, Report No. 3.1(c)-2.

Leiva, G., and Klingner, R.E., 1994, "Behavior and Design of Multi-Story Masonry Walls under In-plane Seismic Loading," *The Masonry Society Journal*, Vol. 13, No. 1.

Matsumura, A., 1988, "Effectiveness f Shear Reinforcement in Fully Grouted Hollow Clay Masonry Walls," *Proceedings of the Fourth Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research*, San Diego, California.

Matsuno, M., Yamazaki, Y., Kaminosono, T., Teshigawara, M., 1987, "Experimental and Analytical Study of the Three Story Reinforced Clay Block Specimen," *Proceedings of the Third Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research*, Tomamu, Hokkaido, Japan.

Mayes, R.L., 1993, Unpublished study in support of the U.S. Coordinated Program for Masonry Building Research Design and Criteria Recommendations for Reinforced Masonry.

Merryman, K.M., Leiva, G., Antrobus, N., and Klingner, 1990, *In-plane Seismic Resistance of Two-story Concrete Masonry Coupled Shear Walls*, U.S. Coordinated Program for Masonry Building Research, Report No. 3.1(c)-1.

Modena, C., 1989, "Italian Practice in Evaluating, Strengthening, and Retrofitting Masonry Buildings," *Proceedings of the International Seminar on Evaluating, Strengthening, and Retrofitting Masonry Buildings*, University of Texas, Arlington, Texas.

Noakowski, Piotr, 1985, *Continuous Theory for the Determination of Crack Width under the Consideration of Bond,* Beton-Und Stahlbetonbau, 7 u..

Noland, J.L., 1990, "1990 Status Report: U.S. Coordinated Program for Masonry Building Research," *Proceedings of the Fifth North American Masonry Conference*, Urbana-Champaign, Illinois.

Ohkubo, M., 1991, *Current Japanese System on Seismic Capacity and Retrofit Techniques for Existing Reinforced Concrete Buildings and Post-Earthquake Damage Inspection and Restoration Techniques*, University of California, San Diego Structural Systems Research Project Report No. SSRP-91/02.

Okada, T., and Kumazawa, F., 1987, "Flexural Behavior of Reinforced Concrete Block Beams with Spirally-Reinforced Lap Splices," *Proceedings of the Third Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research*, Tomamu, Hokkaido, Japan.

Park, R. and T. Paulay, 1975, *Reinforced Concrete Structures*, John Wiley & Sons, New York.

Paulay T., and Priestley, M.J.N, 1992, *Seismic Design of Reinforced Concrete and Masonry Structures*, John Wiley & Sons, New York.

Paulay, T., and Priestley, M.J.N, 1993, "Stability of Ductile Structural Walls," *ACI Structural Journal*, Vol. 90, No. 4.

Paulay, T., Priestley, M.J.N., and Synge, A.J., 1982, "Ductility in Earthquake Resisting Squat Shearwalls," *ACI Journal*, Vol. 79, No. 4, pp.257-269.

Priestley, M.J.N., 1977, "Seismic Resistance of Reinforced Concrete Masonry Shear Walls with High Steel Percentages," *Bulletin of the New Zealand National Society for Earthquake Engineering*, Vol. 10, No. 1, pp.1-16.

Priestley, M.J.N., 1986, "Flexural Strength of Rectangular Unconfined Shear Walls with Distributed Reinforcement," *The Masonry Society Journal*, Vol. 5, No. 2.

Priestley, M.J.N., 1990, *Masonry Wall-Frame Joint Test*, Report No. 90-5, Sequad Consulting Engineers, Solana Beach, California.

Priestley, M.J.N. and Elder, D.M., 1982, *Seismic Behavior of Slender Concrete Masonry Shear Walls*, Dept. of Civil Engineering, University of Canterbury, Christchurch, Research Report ISSN 0110-3326, New Zealand.

Priestley M.J.N., Evison, R.J., and Carr, A.J., 1978, "Seismic Response of Structures Free to Rock on their Foundations," *Bulletin of the New Zealand National Society for Earthquake Engineering*, Vol. 11, No. 3, pp. 141–150.

Priestley, M.J.N., and Hart, G.C., 1989, *Design Recommendations for the Period of Vibration of Masonry Wall Buildings*, Structural Systems Research Project, Report No. SSRP 89/05, University of California at San Diego.

Priestley, M.J.N., and Hon, C.Y., 1985, "Seismic Design of Reinforced Concrete Masonry Moment - Resisting Frames," *The Masonry Society Journal*, Vol. 4., No. 1.

Priestley, M.J.N., and Limin, He, 1990, "Seismic Response of T-Section Masonry Shear Walls," *The Masonry Society Journal*, Vol. 9, No. 1.

Priestley, M.J.N., Verma, R., and Xiao, Y., 1994, "Seismic Shear Strength of Reinforced Concrete Columns," *ASCE Journal of Structural Engineering*, Vol. 120, No. 8.

Sassi, H., and Ranous, R., 1996, "Shear Failure in Reinforced Concrete Walls," *From Experience*, Structural Engineers Association of Southern California, Whittier, California.

Schuller, M.P., Atkinson, R.H., and Borgsmiller, J.T., 1994, "Injection Grouting for Repair and Retrofit of Unreinforced Masonry," *Proceedings of the 10th International Brick and Block Masonry Conference*, Calgary, Alberta, Canada.

Schultz, A.E., 1996 "Seismic Resistance of Partially-Grouted Masonry Shear Walls," *Worldwide Advances in Structural Concrete and Masonry*, A.E. Schultz and S.L. McCabe, Eds., American Society of Civil Engineers, New York.

Seible, F., Hegemier, G.A., Igarashi, A., and Kingsley, G.R., 1994a, "Simulated Seismic-Load Tests on Full- Scale Five-Story Masonry Building," *ASCE Journal of Structural Engineering*, Vol. 120, No. 3.

Seible, F., Kingsley, G.R., and Kürkchübasche, A.G., 1995, "Deformation and Force Capacity Assessment Issues in Structural Wall Buildings," *Recent Developments in Lateral Force Transfer in Buildings – Thomas Paulay Symposium*, ACI SP-157.

Seible, F., Okada, T., Yamazaki, Y., and Teshigawara, M., 1987, "The Japanese 5-story Full Scale Reinforced Concrete Masonry Test — Design and Construction of the Test Specimen," *The Masonry Society Journal*, Vol. 6, No. 2.

Seible, F., Priestley, M.J.N., Kingsley, G.R., and Kürkchübasche, A.G., 1991, *Flexural coupling of Topped Hollow Core Plank Floor Systems in Shear Wall Structures*, Structural Systems Research Project, Report No. SSRP-91/10, University of California at San Diego.

Seible, F., Priestley, M.J.N., Kingsley, G.R., and Kürkchübasche, A.G., 1994b, "Seismic Response of a Five-story Full-scale RM Research Building", *ASCE Journal of Structural Engineering*, Vol. 120, No. 3.

Seible, F., Yamazaki, Y., Kaminosono, T., and Teshigawara, M., 1987, "The Japanese 5-story Full Scale Reinforced Concrete Masonry Test -- Loading and Instrumentation of the Test Building," *The Masonry Society Journal*, Vol. 6, No. 2.

Shing, P.B., Brunner, J., and Lofti, H.R., 1993, "Analysis of Shear Strength of Reinforced Masonry Walls," *Proceedings of the Sixth North American Masonry Conference*, Philadelphia, Pennsylvania.

Shing, P.B., Noland, J.L., Spaeh, H.P., Klamerus, E.W., and Schuller, M.P., 1991, *Response of Single Story Reinforced Masonry Shear Walls to In-plane Lateral Loads*, TCCMAR Report No. 3.1(a)-2.

Shing, P., Schuller, M., Hoskere, V., and Carter, E., 1990a, "Flexural and Shear Response of Reinforced Masonry Walls," *ACI Structural Journal*, Vol. 87, No. 6.

Shing, P., Schuller, M., and Hoskere, V., 1990b, "In-plane Resistance of Reinforced Masonry Shear Walls," *Proceedings of ASCE Journal of the Structural Division*, Vol 116, No. 3.

Soric, Z. and Tulin, L.E., 1987, "Bond in Reinforced Concrete Masonry," *Proceedings Fourth North American Masonry Conference*, Los Angeles, California.

Thurston, S., and Hutchinson, D., 1982, "Reinforced Masonry Shear Walls Cyclic Load Tests in Contraflexure, " *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol. 15, No. 1.

TMS, 1994, *Performance of Masonry Structures in the Northridge, California Earthquake of January 17, 1994*, Richard E. Klingner, ed., The Masonry Society.

Tomazevic, M., and Lutman, M., 1988, "Design of Reinforced Masonry Walls for Seismic Actions," *Brick and Block Masonry*, J.W. DeCourcy, Ed., Elsevier Applied Science, London.

Tomazevic, M., Lutman, M., and Velechovsky, T., 1993, "The Influence of Lateral Load Time-history on the Behavior of Reinforced-Masonry Walls Failing in Shear," *Proceedings of the Sixth North American Masonry Conference*, Philadelphia, Pennsylvania.

Tomazevic, M., and Modena, C., 1988, "Seismic Behavior of Mixed, Reinforced Concrete, Reinforced Masonry Structural Systems," *Brick and Block Masonry*, J.W. DeCourcy, Ed., Elsevier Applied Science, London.

Tomazevic, M., and Zarnic, R., 1985, "The Effect of Horizontal Reinforcement on the Strength and Ductility of Masonry Walls at Shear Failure," *Proceedings of the Seventh International Brick Masonry Conference*, Melbourne, Australia.

Vulcano A., and Bertero, V.V., 1987, *Analytical Models for Predicting the Lateral Response of RC Shear Walls*, University of California, UCB/EERC Report No. 87-19, Berkeley, California.

Weeks, J., Seible, F., Hegemier, G., and Priestley, M.J.N., 1994, *The U.S.-TCCMAR Full-scale Five-story Masonry Research Building Test, Part V - Repair and Retest*, University of California Structural Systems Research Project Report No. SSRP-94/05.

Yamamoto, M., and Kaminosono, T., 1986, "Behavior of Reinforced Masonry Walls with Boundary Beams," *Proceedings of the Second Meeting of the Joint Technical Coordinating Committee on Masonry Research*, Keystone, Colorado.

Yamazaki, Y., F. Seible, H., Mizuno, H., Kaminosono, T., Teshigawara, M., 1988b, "The Japanese 5-story Full Scale Reinforced Concrete Masonry Test -- Forced Vibration and Cyclic Load Test Results," *The Masonry Society Journal*, Vol. 7, No. 1.

Yamazaki, Y., Kaminosono, T., Teshigawara, M., and Seible, F., 1988a, "The Japanese 5-story Full Scale Reinforced Concrete Masonry Test -- Pseudo Dynamic and Ultimate Load Test Results," *The Masonry Society Journal*, Vol. 7, No. 2.

Young, J.M., and Brown, R.H., 1988, *Compressive Stress Distribution of Grouted Hollow Clay Masonry Under Stain Gradient*, U.S. Coordinated Program for Masonry Building Research, Report No. 1.2b-1.

4. Unreinforced Masonry

4.1 Commentary and Discussion

4.1.1 Hysteretic Behavior of URM Walls Subjected to In-Plane Demands

A search of the available literature was performed to identify experimental and analytical research relevant to unreinforced masonry bearing-wall damage. Because URM buildings have performed poorly in past earthquakes, there is an extensive amount of anecdotal information in earthquake reconnaissance reports; there have also been several studies that took a more statistical approach and collected damage information in a consistent format for a comprehensive population of buildings. These studies help to confirm the prevalence of the damage types listed in FEMA 306, and they help to indicate the intensity of shaking required to produce certain damage types.

The proposed methodology for this document, however, requires moving beyond anecdotal and qualitative discussions of component damage and instead obtaining quantitative information on force/displacement relationships for various components. The focus of research on URM buildings has been on the in-plane behavior of walls. Most of the relevant research has been done in China, the former Yugoslavia, Italy, and the United States. This stands in contrast to the elements in URM buildings that respond to ground shaking with essentially brittle or force-controlled behavior: parapets, appendages, wall-diaphragm ties, out-of-plane wall capacity, and, possibly, archaic diaphragms such as brick arch floors. While there has been very little research on most of these elements, it is less important because performance of these elements is not deformation-controlled.

Unfortunately, research on in-plane wall behavior is rarely consistent—materials, experimental techniques, modes of reporting, and identified inelastic mechanisms all vary widely. Placing the research in a format consistent with FEMA 273 and this project's emphasis on components, damage types, hysteresis curves, nonlinear force/displacement relationships, and performance levels is difficult. Almost no experimental tests have been done on damaged URM walls; typically, tests were done on undamaged walls and stopped. In some cases, the damaged wall was repaired and retested. Most of the research does not provide simple predictive equations for strength and stiffness (particularly post-elastic stiffness); when analysis has been done, it has usually used fairly sophisticated finite element modelling techniques.

Hysteresis loops for in-plane wall behavior are shown on the following pages, Sections 4.1.1.1 to 4.1.1.6, organized by behavior mode. Research shows that the governing behavior mode depends upon a number of variables including material properties, aspect ratio, and axial stress. To aid in comparing the curves, basic data given in the research report are provided, including the average compressive strength of prism tests and the masonry unit, the pier aspect ratio, the nominal axial stress, and whether the specimen was free to rotate at the top (cantilever condition) or was fixed (double-curvature condition). For many of the specimens, independent calculations have been carried out for this document to allow comparison between the evaluation procedure predictions in Section 7.3 of FEMA 306 and the actual experimental results. Predictions using FEMA 273 are also noted. In several cases, engineering judgment has been exercised to make these calculations, since not all of the necessary information is available. Material properties that were assumed for the purposes of the calculation are identified. It is expected that predicted results could vary significantly if different assumptions are made. In addition, the experimental research in URM piers is difficult to synthesize for several reasons:

- Some researchers do not report a measure of bed-joint sliding-shear strength. Others use triplet tests rather than in-place push tests to measure bed-joint sliding capacity. Comparisons between triplet tests and in-place push tests are not well established. Several different assumptions were investigated for this project, and the approach shown below was found to correlate best with the data.
- Descriptions of cracking can be inconsistent and overly vague. Diagonal cracking, for example, is often reported, but it can be unclear if the report refers to diagonal tension cracking, toe crushing with diagonally-oriented cracks, or stair-stepped bed-joint sliding.
- Observed damage is often not linked to points on the force/displacement hysteresis loops.
- Final drift values are not always given; when they are, it is often unclear why the test was stopped and

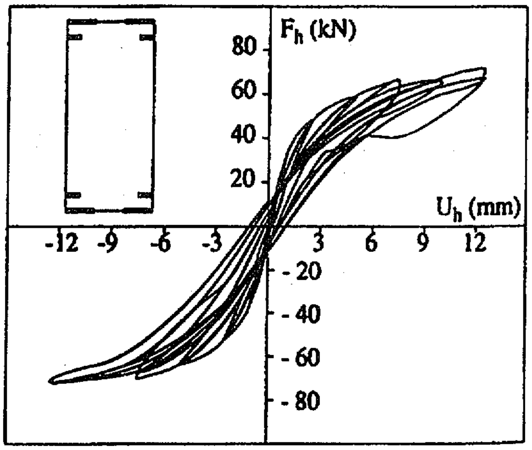
whether additional stable deformation capacity remained.

- In many tests, the applied axial load varies significantly from the desired nominal value at different times during the test. Thus, lateral capacities can be affected.

4.1.1.1 Rocking

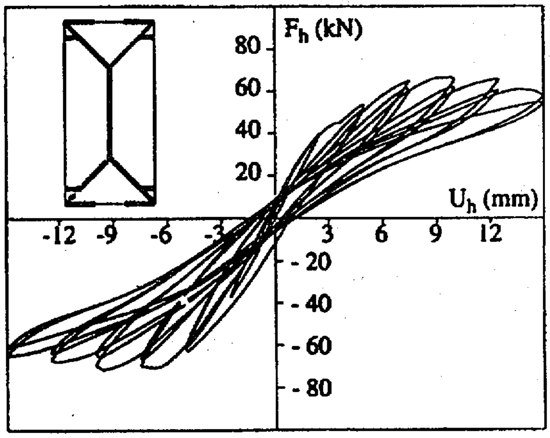
Reference: Anthoine et al. (1995)
Specimen: High wall, first run
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m=6.2 MPa, brick f'_m=16 MPa
L/h_{eff}=1m/2m= 0.5
Nominal f_a=0.60 MPa
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.23+0.57f_a) MPa
v_{me2}=(0.75/1.5)*(0.57f_a) MPa
Calculated Values (kN):
V_r=68 V_{tc}=65
V_{bjs1}=73 V_{bjs2}=43
V_{dt1}=85 V_{dt2}=130
FEMA 273 Predicted Mode: Toe crushing
ATC-43 Predicted Behavior: Rocking at 68 kN with drift "d"=0.8%
Actual Behavior: Rocking at 72 kN with test stopped at 0.6%. Slight cracks at mid-pier. Axial load increased for second run (see below).

- There is no direct test for f'_{dt1}. FEMA 273 equations use v_{me} for f'_{dt}. This gives the value for V_{dt1}. As an additional check, 1/30th of the value of flat-wise compressive strength of the masonry units was also used; this results in the value for V_{dt2}.



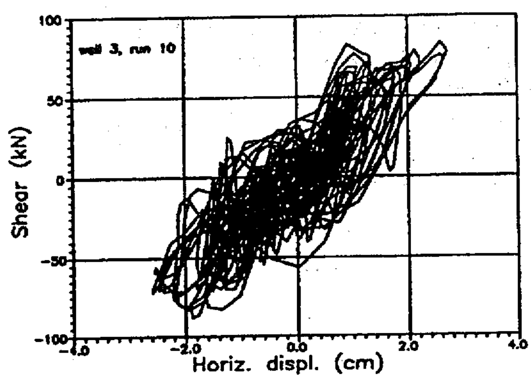
Hysteretic response of the high wall, first run.

Reference: Anthoine et al. (1995)
Specimen: High wall, second run
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m=6.2 MPa, brick f'_m=16 MPa
L/h_{eff}=1m/2m= 0.5
Nominal f_a= 0.80 MPa
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.23+0.57f_a) MPa
v_{me2}=(0.75/1.5)*(0.57f_a) MPa
Calculated Values (kN):
V_r=90 V_{tc}=82
V_{bjs1}=85 V_{bjs2}=58
V_{dt1}=104 V_{dt2}=141
FEMA 273 Predicted Mode: Toe crushing
ATC-43 Predicted Behavior: Same as FEMA 273
Actual Behavior: Rocking, then stair-stepped bed-joint sliding at a drift of 0.75%



Hysteretic response of the high wall, second run.

Reference: Magenes & Calvi (1995)
Specimen: 3, runs 7-12
Material: Brick
Loading: Shaketable
Provided Information:
Prism f'_m=8.6 MPa, brick f'_m=18.2 MPa
L/h_{eff}=1m/2m = 0.5
Nominal f_a= 0.63 MPa
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(1.15+0.57f_a) MPa
v_{me2}=(0.75/1.5)*(0.57f_a) MPa
Calculated Values (kN):
V_r=71 V_{tc}=70
V_{bjs1}=189 V_{bjs2}=45
V_{dt1}=171 V_{dt2}=145
FEMA 273 Predicted Mode: Toe crushing
ATC-43 Predicted Behavior: Rocking at 71 kN with drift "d" = 0.8%.
Actual Behavior: Rocking at 87 kN with drift of 1.3% in run 10.



Shear-displacement curve characterized by rocking (wall 3, run 10). The figure does not show final runs 11 and 12.

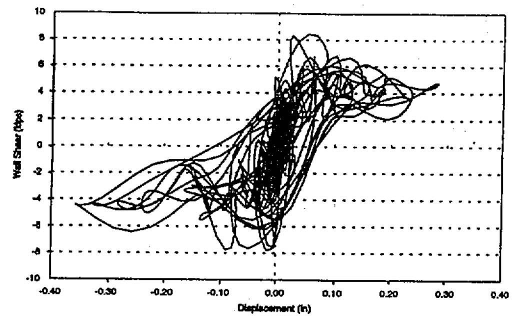
Reference: Costley & Abrams (1996)
Specimen: S1 Door Wall
Material: Brick
Loading: 3/8th-scale building on shaketable
Provided Information:
Prism f'_m=1960 psi, brick f'_m=6730 psi
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.75*361+f_a) psi
v_{me2}=(0.75/1.5)*(f_a) psi
Outer Piers:
L/h_{eff}=1.44ft/2.67ft =0.54
Nominal f_a= 33 psi
Calculated Values (kips):

V_r=1.0	V_{tc}=1.1
V_{bjs1}=9.7	V_{bjs2}=1.1
V_{dt1}=7.2	V_{dt2}=10.3

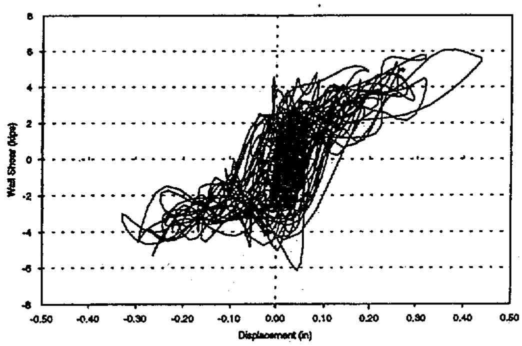
Inner Pier:
L/h_{eff}=0.79ft/1.50ft =0.53
Nominal f_a= 40 psi
Calculated Values (kips):

V_r=2.7	V_{tc}=2.9
V_{bjs1}=15.3	V_{bjs2}=1.8
V_{dt1}=14.3	V_{dt2}=20.4

FEMA 273 Predicted Behavior of Wall Line: Rocking at 4.7 kips with inner-pier drift "d"=0.5%
ATC-43 Predicted Behavior of Wall Line: Same as FEMA 273
Actual Behavior of the Wall Line:
Run 14: Rocking up to 8 kips, then stable at 4-6 kips. Drift up to 1.1%.
Run 15: Rocking at 4-6 kips with drift up to 1.3%



Door-wall shear vs. first-level door-wall displacement from Test Run 14



Door-wall shear vs. first-level door-wall displacement from Test Run 15

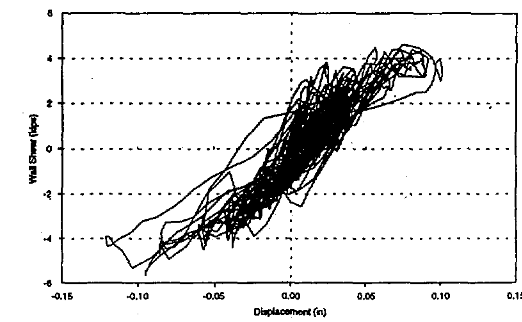
Reference: Costley & Abrams (1996)
Specimen: S2 Door Wall
Material: Brick
Loading: 3/8th-scale building on shaketable
Provided Information:
Prism f'_m=1960 psi, brick f'_m=6730 psi
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.75*361+f_a) psi
v_{me2}=(0.75/1.5)*(f_a) psi
Outer Piers:
L/h_{eff}=0.79ft/2.67ft =0.30
Nominal f_a= 40 psi
Calculated Values (kips):

V_r=0.4	V_{tc}=0.4
V_{bjs1}=5.5	V_{bjs2}=0.7
V_{dt1}=4.1	V_{dt2}=5.7

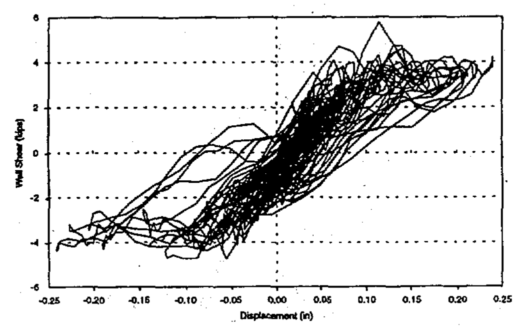
Inner Piers:
L/h_{eff}=1.12ft/2.67ft =0.42
Nominal f_a= 48 psi
Calculated Values (kips):

V_r=0.9	V_{tc}=1.0
V_{bjs1}=7.9	V_{bjs2}=1.2
V_{dt1}=6.1	V_{dt2}=8.2

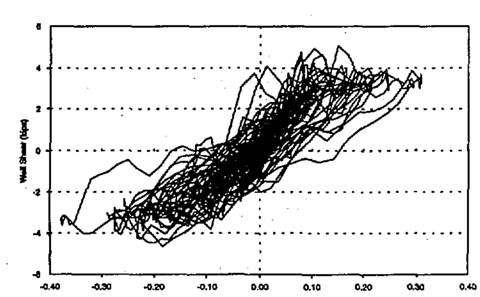
FEMA 273 Predicted Behavior of Wall Line: Rocking at 2.6 kips with inner-pier drift "d"=1.0%
ATC-43 Predicted Behavior of Wall Line: Same as FEMA 273
Actual Behavior of the Wall Line:
Run 22: Rocking at 4 kips, up to a 0.3% drift
Run 23: Rocking at 4 kips, up to a 0.8% drift
Run 24: Rocking at 4 kips, up to a 1.1% drift



Door-wall shear vs. first-level door-wall displacement from Test Run 22



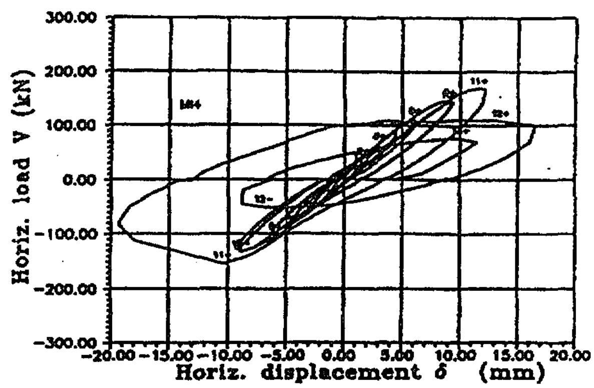
Door-wall shear vs. first-level door-wall displacement from Test Run 23



Door-wall shear vs. first-level door-wall displacement from Test Run 24

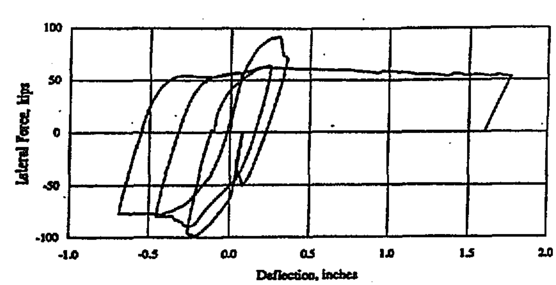
4.1.1.2 Bed-joint Sliding

Reference: Magenes & Calvi (1992)
Specimen: MI4
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m=7.9 MPa, brick f'_m=19.7 MPa
L/h_{eff}=1.5m/3m = 0.5
Nominal f_a= 0.69 MPa
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.206+0.813f_a) MPa
v_{me2}=(0.75/1.5)*(0.813f_a) MPa
Calculated Values (kN):
V_r=177 V_{tc}=172
V_{bjs1}=219 V_{bjs2}=160
V_{dt1}=245 V_{dt2}=360
FEMA 273 Predicted Behavior: Toe crushing at 172 kN
ATC-43 Predicted Behavior: Rocking at 177kN with drift "d" = 0.8%
Actual Behavior: Stair-stepped bed-joint sliding at 153 kN with a final drift of 0.6%



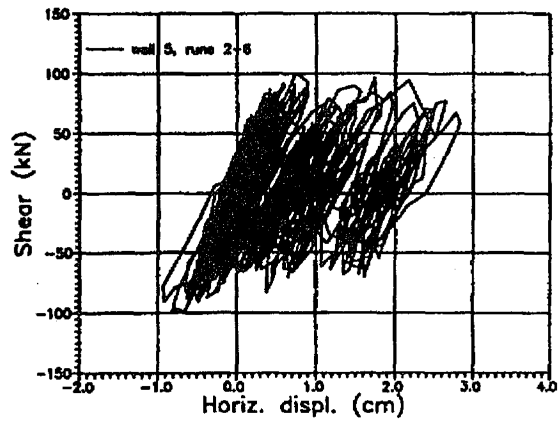
Specimen MI4

Reference: Abrams & Shah (1992)
Specimen: W1
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m=911 psi, brick f'_m=3480 psi
L/h_{eff}=12ft/6ft = 2
Nominal f_a= 75 psi
Cantilever conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.75*100+f_a) psi
v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
V_r=76 V_{tc}=74
V_{bjs1}=84 V_{bjs2}=42
V_{dt1}=149 V_{dt2}=167
FEMA 273 Predicted Behavior: Toe crushing at 74 kips
ATC-43 Predicted Behavior: Flexural cracking/toe crushing/bed-joint sliding with a peak load of 74 kips with "d" drift of 0.4%
Actual Behavior: Bed-joint sliding at 92 kips with test stopped at a drift of 2.4%.



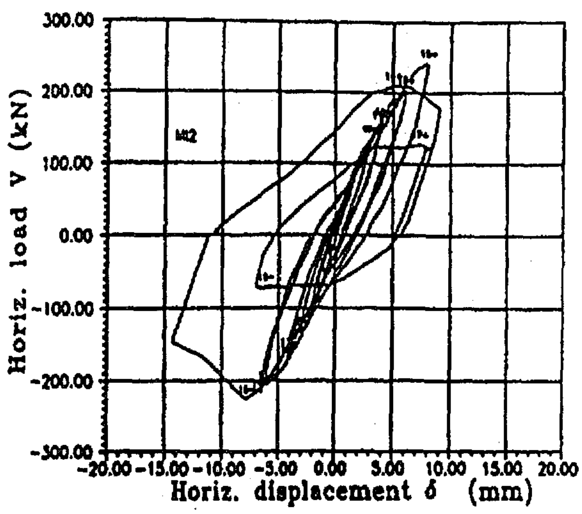
Test Wall W1

Reference: Magenes & Calvi (1995)
Specimen: 5
Material: Brick
Loading: Shaketable
Provided Information:
Prism f'_m=6.2 MPa, brick f'_m=16 MPa
L/h_{eff}=1m/1.35m = 0.74
Nominal f_a= 0.63 MPa
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.23+0.57f_a) MPa
v_{me2}=(0.75/1.5)*(0.57f_a) MPa
Calculated Values (kN):
V_r=105 V_{tc}=102
V_{bjs1}=74 V_{bjs2}=45
V_{dt1}=97 V_{dt2}=160
FEMA 273 Predicted Behavior: Bed-joint sliding at 74 kN with "d" drift of 0.4%
ATC-43 Predicted Behavior: Same as FEMA 273
Actual Behavior: Flexural cracking then horizontal and stepped bed-joint sliding with peak load of 114 kN



Shear-displacement curve characterized by rocking and sliding (wall 5, runs 2-6). The figure does not show final run 7.

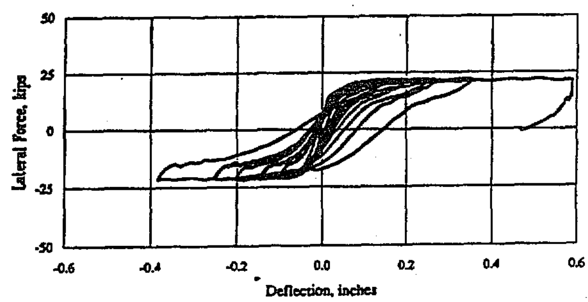
Reference: Magenes & Calvi (1992)
Specimen: MI2
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m=7.9 MPa, brick f'_m=19.7 MPa
L/h_{eff}=1.5m/2m = 0.74
Nominal f_a= 0.67 MPa
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.206+0.813f_a) MPa
v_{me2}=(0.75/1.5)*(0.813f_a) MPa
Calculated Values (kN):
V_r=257 V_{tc}=251
V_{bjs1}=213 V_{bjs2}=155
V_{dt1}=267 V_{dt2}=399
FEMA 273 Predicted Behavior: Bed-joint sliding at 213 kN with "d" drift of 0.4%.
ATC-43 Predicted Behavior: Same as FEMA 273
Actual Behavior: Horizontal bed-joint sliding at top course, then stair-stepped bed-joint sliding with a peak load of 227 kN and drift of 0.7%



Specimen MI2

4.1.1.3 Rocking/Toe Crushing

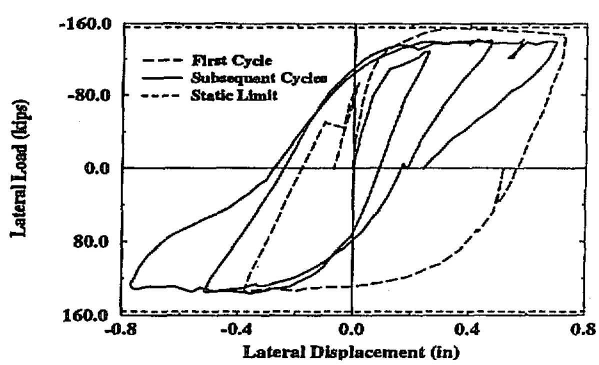
Reference: Abrams & Shah (1992)
Specimen: W3
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m= 911 psi, brick f'_m= 3480 psi
L/h_{eff}= 6ft/6ft =1.0
Nominal f_a= 50 psi
Cantilever conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.75*100+f_a) psi
v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
V_r= 12.6 V_{tc}=12.9
V_{bjs1}=35 V_{bjs2}=14
V_{dt1}=69 V_{dt2}=78
FEMA 273 Predicted Behavior: Rocking at 12.6 kips with drift "d"=0.4%
ATC-43 Predicted Behavior: Same as FEMA 273
Actual Behavior: Rocking at 20 kips then toe crushing at drift of 0.8%



Test Wall W3: Measured relation between lateral force and deflection.

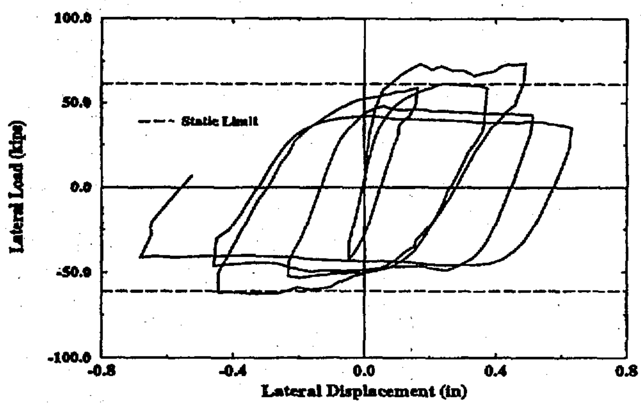
4.1.1.4 Flexural Cracking/Toe Crushing/Bed-Joint Sliding

Reference: Manzouri et al. (1995)
Specimen: W1
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m= 2000 psi, brick f'_m= 3140 psi
L/h_{eff}= 8.5ft/5ft =1.7
Nominal f_a= 150 psi
Cantilever conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.75*85+f_a) psi
v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
V_r= 152 V_{tc}=151
V_{bjs1}=156 V_{bjs2}=99
V_{dt1}=235 V_{dt2}=172
FEMA 273 Predicted Behavior: Toe crushing at 151 kips.
ATC-43 Predicted Behavior: Flexural cracking/toe crushing/bed-joint sliding with a 151 kip peak load, 99 kip load for "c" and a "d"drift of 0.4%.
Actual Behavior: Flexural cracking at 88 kips, toe crushing then bed-joint sliding at 156 kips, with a final drift of 1.3%



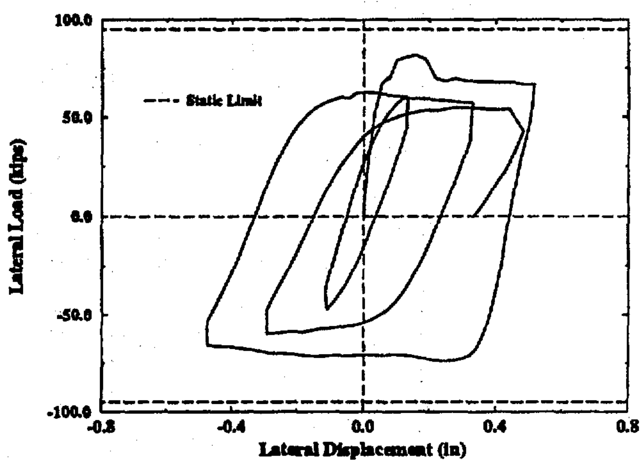
Specimen W1

Reference: Manzouri et al. (1995)
Specimen: W2
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m= 2200 psi, brick f'_m= 3140 psi
L/h_{eff}= 8.5ft/5ft =1.7
Nominal f_a= 55 psi
Cantilever conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.75*85+f_a) psi
v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
V_r= 56 V_{tc}=60
V_{bjs1}=93 V_{bjs2}=36
V_{dt1}=124 V_{dt2}=171
FEMA 273 Predicted Behavior: Rocking at 56 kips.
ATC-43 Predicted Behavior: Flexural cracking/toe crushing at 60 kips.
Actual Behavior: Flexural cracking at 31 kips, toe crushing at 68 kips, diagonal cracking at 62 kips, then bed-joint sliding at 52 kips and below, with a final drift of 1.2%



Specimen W2

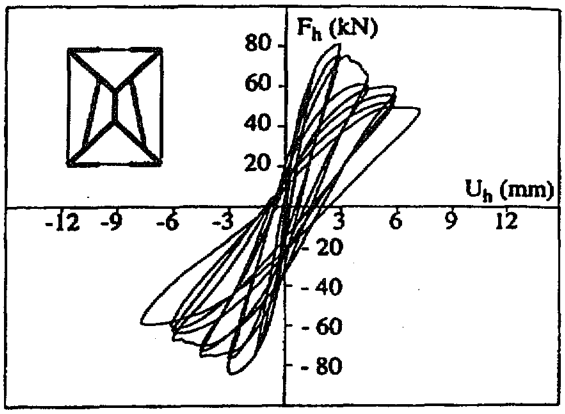
Reference: Manzouri et al. (1995)
Specimen: W3
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m= 2600 psi, brick f'_m= 3140 psi
L/h_{eff}= 8.5ft/5ft =1.7
Nominal f_a= 85 psi
Cantilever conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.75*85+f_a) psi
v_{me2}=(0.75/1.5)*(f_a) psi
Calculated Values (kips):
V_r= 86 V_{tc}=91
V_{bjs1}=113 V_{bjs2}=56
V_{dt1}=159 V_{dt2}=187
FEMA 273 Predicted Behavior: Rocking at 86 kips.
ATC-43 Predicted Behavior: Flexural cracking/toe crushing/bed-joint sliding with a 91 kip peak load, 56 kip load for "c" and a "d"drift of 0.4%.
Actual Behavior: Flexural cracking at 55 kips, toe crushing at 80 kips, then bed-joint sliding at 80 kips, reducing to 56-62 kips, with some final toe crushing up to final drift of 0.8%



Specimen W3

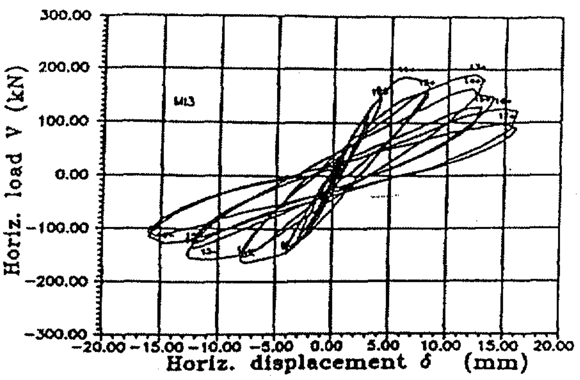
4.1.1.5 Flexural Cracking/Diagonal Tension

Reference: Anthoine et al. (1995)
Specimen: Low Wall
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m=6.2 MPa, brick f'_m=16 MPa
L/h_{eff}=1m/1.35m= 0.74
Nominal f_a= 0.60 MPa
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.23+0.57f_a) MPa
v_{me2}=(0.75/1.5)*(0.57f_a) MPa
Calculated Values (kN):
V_r=100 V_{tc}=96
V_{bjs1}=73 V_{bjs2}=43
V_{dt1}=94 V_{dt2}=144
FEMA 273 Predicted Behavior: Bed-joint sliding at 73 kips with "d" drift of 0.4%
ATC-43 Predicted Behavior: Same as FEMA 273
Actual Behavior: Flexural cracking then diagonal tension cracking with a peak load of 84 kN and a final drift of 0.5%



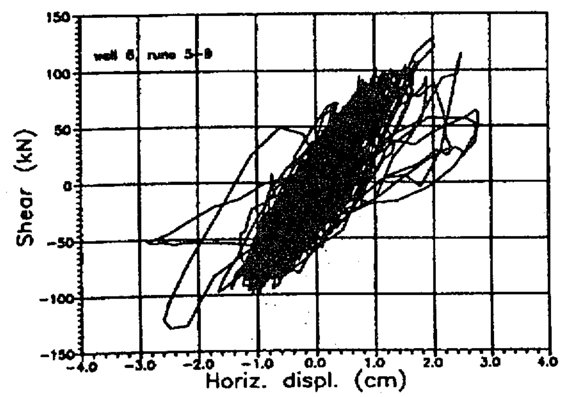
Hysteretic response of the low wall.

Reference: Magenes & Calvi (1992)
Specimen: MI3
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m=7.9 MPa, brick f'_m=19.7 MPa
L/h_{eff}=1.5m/3m = 0.5
Nominal f_a= 1.245 MPa
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.206+0.813f_a) MPa
v_{me2}=(0.75/1.5)*(0.813f_a) MPa
Calculated Values (kN):
V_r=319 V_{tc}=275
V_{bjs1}=347 V_{bjs2}=288
V_{dt1}=406 V_{dt2}=427
FEMA 273 Predicted Behavior: Toe crushing
ATC-43 Predicted Behavior: Flexural cracking/ diagonal tension at 275 kN
Actual Behavior: Flexural cracking then diagonal tension cracking with a peak load of 185 kN and a final drift of 0.5%



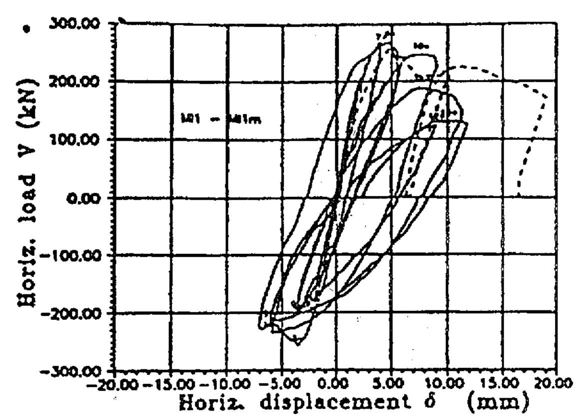
Specimen MI3

Reference: Magenes & Calvi (1995)
Specimen: 8
Material: Brick
Provided Information:
Prism f'_m=6.2 MPa, brick f'_m=16 MPa
L/h_{eff}=1m/2m = 0.5
Nominal f_a= 1.11 MPa
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.23+0.57f_a) MPa
v_{me2}=(0.75/1.5)*(0.57f_a) MPa
Calculated Values (kN):
V_r=125 V_{tc}=109
V_{bjs1}=108 V_{bjs2}=79
V_{dt1}=137 V_{dt2}=171
FEMA 273 Predicted Behavior: Bed-joint sliding or toe crushing.
ATC-43 Predicted Behavior: Bed-joint sliding or flexural cracking/diagonal tension at 108-109 kN
Actual Behavior: Flexural cracking then diagonal tension cracking with a peak load of 129 kN and a final drift of 0.8-1.3%



Brittle collapse due to diagonal cracking (wall 8, runs 5-9)

Reference: Magenes & Calvi (1992)
Specimen: MI1
Material: Brick
Loading: Reversed quasistatic cyclic
Provided Information:
Prism f'_m=7.9 MPa, brick f'_m=19.7 MPa
L/h_{eff}=1.5m/2m= 0.75
Nominal f_a= 1.123 MPa
Fixed-fixed end conditions
Assumed Values:
v_{me1}=(0.75/1.5)*(0.206+0.813f_a) MPa
v_{me2}=(0.75/1.5)*(0.813f_a) MPa
Calculated Values (kN):
V_r=432 V_{tc}=383
V_{bjs1}=319 V_{bjs2}=260
V_{dt1}=415 V_{dt2}=462
FEMA 273 Predicted Behavior: Bed-joint sliding at 319 kN with drift "d"=0.4%
ATC-43 Predicted Behavior of Wall Line: Same as FEMA 273
Actual Behavior: Flexural cracking then diagonal tension at 259 kN, with maximum drift of 0.6%



Test on wall MI1 and MI1m (dashed line); h = 2m.